DESIGN STATEMENT
DRAINAGE CALCULATIONS
&
HYDRAULIC ANALYSIS

UNIGLOBE INVESTMENTS, LLC

91 & 93 MEADOWBROOK LANE
MANSFIELD CONNECTICUT

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OVERVIEW & SITE LOCATION

The project proposed entails the development of a modest residential development on a 4.6 acre parcel on the south side of Meadowbrook Lane in Mansfield, Connecticut adjacent the previously approved Whispering Glen development. The proposed development will consist of four new buildings, three of which will contain five units each of housing, one which will contain three units. The units will be a mixture of one-bedroom units and two-bedroom units with a total of 60 bedrooms in the development.

Access to the development will be by way of a 95' long boulevard from Meadowbrook Lane, and an additional access road from the Whispering Glenn development.

The parcel on which this development is proposed is currently in an (R-20) Residential 20 Zone. This application will include a request for a zone change to a Planned Residential Zone.

GEOGRAPHY

The subject site is located on a relatively flat sandy area, with a gentle 4-5% slope from east to west and which also drains to the south through an existing well-defined gulley along the eastern property line. The far west and south side of the site is bound by an un-named brook which empties into the Conantville Brook about 250' east of the subject parcel. The Conantville Brook in turn empties into Sawmill Brook just south of the Eastbrook Mall, and Sawmill Brook continues on to the south and meets the Natchaug River at the north end of Phillip Lauter Park in Willimantic.

According to the Natural Resources Conservation Service Web Soil Survey, the soils on the site are primarily Gloucester gravelly sandy loam on steeps slopes, and Canton and Charleton soils on 3 to 8% slopes, which are typified as well-drained, sandy and gravelly sandy loams, with moderately high to high transmissivity, and greater than 80 inches to restrictive features. These soils belong to Hydrologic Soil Group B.

EXISTING CONDITIONS / PROPOSED CONDITIONS

Most of the property to be affected by the proposed development has been previously cleared and is relatively open sandy ground. There currently exists on the parcel a wood-framed residential building, a mobile home and a number of out buildings. The proposed plan calls for an eighteen–unit development with a total of 60 bedrooms, and paved parking for 77 vehicles. Obviously, the amount of impervious coverage on the site will increase dramatically.

The stormwater management plan that we have designed will mitigate the effects of that change by utilizing the hydrologic qualities of the underlying soils in conjunction with a network of drywells and catch basins to absorb what would typically be a significant increase in stormwater run-off from the site. By breaking up the drainage areas into small enough segments, and routing the stormwater into drywells that provide both storage volume and surface interface area with surrounding soils, we are able to provide a system capable of handling any storm event up to a 50-year storm without allowing any run-off from the developed portion of the site.

In the event that the stormwater system fails to contain a storm event, and there is run-off from the site, that run-off will be routed to one of two stabilized areas: one at the northwest corner of the project near the entrance road, and one at the southeast corner of the project where stormwater run-off from the site currently discharges to the existing stream.

DRAINAGE

As this is a residential development, the Mansfield Public Works Standards require that we design the driveway, roadway and general drainage system to meet requirements for a 10-year storm intensity. We have met those requirements and beyond.

Driveway area drainage consists of seven catch basins only, and five of those catch basins are in sag locations. Peak discharge of run-off from the drainage areas contributing to the catch basins was determined using both Rational and the SCS hydrograph methods of determination. The Rational Method was used initially because the peak discharge from that method is typically a bit more conservative (greater peak) than the SCS Method. The results of the Rational Method calculations were used to examine how much stormwater bypass we could expect from the two catch basins not in a sag configuration, and to determine the maximum expected peak flows to the catch basins in located in sags. In all instances we determined that the basins would perform satisfactorily for 10 and 25-year storm events. Drainage calculations are contained in Appendix A.

In the event that flows were to exceed those anticipated, overflow from the proposed drainage system would run to areas adjacent to established streams or channels and any negative impact would be minimized.

HYDROLOGY

The point of interest in our comparison of present conditions to post-development stormwater conditions is the point on the southeast corner of the property where the un-named brook leaves the site. Since the brook is the receptor for all existing stormwater run-off from the site, and will remain the receptor under improved conditions, it is a logical point at which to measure the impact of the development on the site.

The stormwater management plan for the proposed design utilizes a series of drywells and lawn drains in small defined areas to capture roof run-off and stormwater and introduce it to the ground in small segments rather than gathering it in a large detention or retention area. In fact, we feel the design before the Commission will result in a significant decrease in stormwater run-off from the site and will help recharge the groundwater that eventually feeds the un-named brook.

We used the Hydraflow Hydrographs Extension for AutoCAD program to model existing conditions and proposed storm sewers and to analyze the capacity of the system for the 2-year, 10-year, 25-year, and 50-year storm events. Models were constructed utilizing SCS run-off hydrographs to generate stormwater volumes, and drywells were modeled as small ponds with a specified exfiltration rate.

The following assumptions and parameters were used as input data in our Hydraflow model of existing conditions:

A curve number (CN) of 61 was used for open space, lawns and parks (existing conditions) in hydrologic soil group B; CN of 98 was used for impervious areas (only 0.10 acre under existing conditions); existing time of concentration was calculated to be 29.9 minutes based in large part on an overland flow distance of 150 feet on slopes of 4.7%, and a manning's coefficient of 0.40 (lawn area), and a time in brook of about 8 minutes.

As mentioned above, the proposed stormwater design is based on the theory that we can capture all the stormwater from impervious areas and introduce it to a very permeable subsoil for groundwater recharge. That being the case, the stormwater run-off expected from the site under the proposed conditions would be constrained to run-off from those areas outside of the area "captured" by curbing and the proposed recharge system.

For the proposed conditions model we used a CN of 65 with the assumption that the area that would still be contributing to off-site run-off would generally be on the steeper slopes surrounding the development. We also reduced the time of concentration for proposed conditions to 15 minutes realizing that the run-off from the most remote areas of the site would be intercepted on site. A comparison of the calculated run-off from the site under existing conditions and proposed conditions follows:

STORM EVENT	EXISTING COND.	DEVELOPED COND.
2-YR	1.3 cfs	1.1 cfs
10-YR	4.5 cfs	3.2 cfs
25-YR	6.2 cfs	4.3 cfs
50-YR	8.0 cfs	5.5 cfs

Before starting our hydrographic model of the site, we conducted soil testing on the site. We excavated four large pits on the site, physically examined the soils, and had Materials Testing, Inc. (MTI), take in-situ samples of the material to conduct permeability testing on the material. We found the majority of the material to be med to coarse sand, and sand and gravel, with one exception being in the lower, southern end of test pit #2 where the material was much finer, siltier. The test results from MTI indicated a permeability rate of 10.5 ft./day for the sample taken from the bottom test hole #2 (which we feel was not a particularly representative sample of the site as a whole), and ranged from 68.7 ft./day to 106.2 ft./day in the other three pits, and an overall average of the test results was about 65 ft./day. A copy of the MTI test data is included in Appendix B.

In modeling the proposed run-off capture and disposal plan for the on-site drainage, we modeled a yard drain (with integral 6'x 6' drywell) as a pond with an incremental storage volume and an incremental rate of discharge related to depth of water in the well. Our calculations show that the storage volume of a 6'x6' drywell with 2' of stone around it will provide about 290 cubic feet of storage over the 6 foot depth, and in increments of 48.4 cubic feet per foot elevation. We then calculated the wetted surface area provided at the interface with the stone and, based on an assumed exfiltration rate of 60 feet per day, determined an initial exfiltration rate of about 0.10 cfs for one foot of depth, with an incremental increase of about 0.02 cfs per foot depth. We ran more than 30 hydrographs to determine the appropriate size areas of capture and to ensure some factor of safety.

Because there are so many small drainage areas, and variations in the percentages of impervious surfaces associated with the areas, we have simplified the smaller areas by treating the areas as if they were entirely impermeable surfaces. In doing so, we have found that a 6'x6' drywell (with 2' of stone) will accommodate approximately 3300 sf having a run-off coefficient of CN=98. If we change our permeability factor to 30 ft./day, we can still accommodate a 10-year storm event, but slightly overtops in a 25-year event. Only two drainage areas feeding drywells are greater than 3300 sf, and those areas (DW#12 and DW#13) are connected by a level perforated pipe to two other drywells (DW9 and DW#14) so that the average area for each of the drywells is less than 3300 sf.

Drywell volume calculations and a typical hydrograph for an impervious area of 3300 sf are included in Appendix C.